US Army Corps
of Engineers ®
Portland District

September 2003 Progress Report: Fern Ridge Embankment

EXECUTIVE SUMMARY

Significant distress has been observed at the Fern Ridge Embankment since August, 2002. The distress includes depressions on the downstream slope of the embankment, deterioration of the 60-year-old drainage system, increases in flow rates from the drainage system during heavy rain events, and seepage emerging from the downstream slope of the embankment during the rainy season. The observed distress has led to two primary concerns: 1) internal erosion (embankment and/or foundation) into the failing drainage system and 2) downstream slope instability due to high water levels in the embankment.

Investigations have been/will be performed and instruments have been/will be installed to gather information necessary to understand the causes of the observed distress, operate the dam safely, and design repair alternatives.

An interim plan has been developed to operate the project safely until it can be repaired. The interim plan involves a conditional operating plan for the reservoir elevation based on piezometric water levels in the embankment, which will be monitored using thirty-eight automated piezometers. The conditional operating plan may be altered in the future if further investigations indicate more favorable conditions, or if interim measures are constructed to improve downstream slope stability. The interim plan also involves an event alert system to prevent the initiation/progression of internal erosion into the failing drainage system.

A future letter report will include a seepage analysis, stability analysis, and discussion on the causes of the distress.

FERN RIDGE DAM
PERTINENT DATA

GENERAL	
Drainage area, square miles	275
Pool elevations, feet	
Minimum flood control pool	353.0
Maximum conservation pool	373.5
Maximum full pool	375.1
RESERVOIR	
Total storage, maximum full pool, acre-feet	111,400
Total storage, maximum conservation pool, acre-feet	97,300
Minimum flood control pool, acre-feet	2,800
DAM	
Type: Earthfill with concrete gravity non-overflow section and concrete gated spillway	
Total crest length, feet	6,610
Dike No. 1 crest length, feet	915
Dike No. 2 crest length, feet	4,145
Crest elevation, feet	382
Crest width, feet	20
Maximum height, feet	44
Freeboard (above maximum full pool), feet	7
SPILLWAY	
Type: Concrete gravity, gate controlled	
Total length, feet	248
Net length, feet	204
Gates	6, tainter
Size of gates, feet	34 by 17.7
Weir crest elevation, feet	358.5
Top of gates, elevation, feet	375.5
Capacity at maximum conservation pool, El. 373.5 ft, ft^3/s	41,400
Capacity at maximum full pool, El. 375.1 ft, ft^3/s	47,200

OUTLET WORKS

Type: Sliding gate	
Type gates	4 sliding gates and 1 sluice gate
Gate size	
Outlet gates, feet	6.75 by 9.67
Sluice gate, feet	3 by 3
Invert elevations, feet	
Outlet gates	339.0
Sluice gate	341.5
Discharge capacity, ft^3/s	
Outlet gates: Minimum flood control pool, El. 353	4,560
Outlet gates: Maximum conservation pool, El. 373.5	8,260
Outlet gates: Maximum full pool, El. 375.1	8,440
Sluice gate: Minimum flood control pool, El. 353	136
Sluice gate: Maximum conservation pool, El. 373.5	254
Sluice gate: Maximum full pool, El. 375.1	261

COYOTE CREEK DIVERSION SYSTEM

Two 10-inch gated intake openings at outlet structure	
One 10-inch diameter steel pipe	
Capacity at minimum flood control pool, ft^3/s	3
Capacity at maximum conservation pool, ft^3/s	8.4

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SECTION I – INTRODUCTION

1.01. Purpose. This report describes the distress observed at Fern Ridge Dam since August, 2002, the concerns raised by the distress, investigations conducted, investigations planned, instruments installed to date, future instrumentation needs, and the interim plan for operating the dam until it can be repaired. A future letter report will include a seepage analysis, stability analysis, and discussion on causes of the distress.

1.02. Background Information.

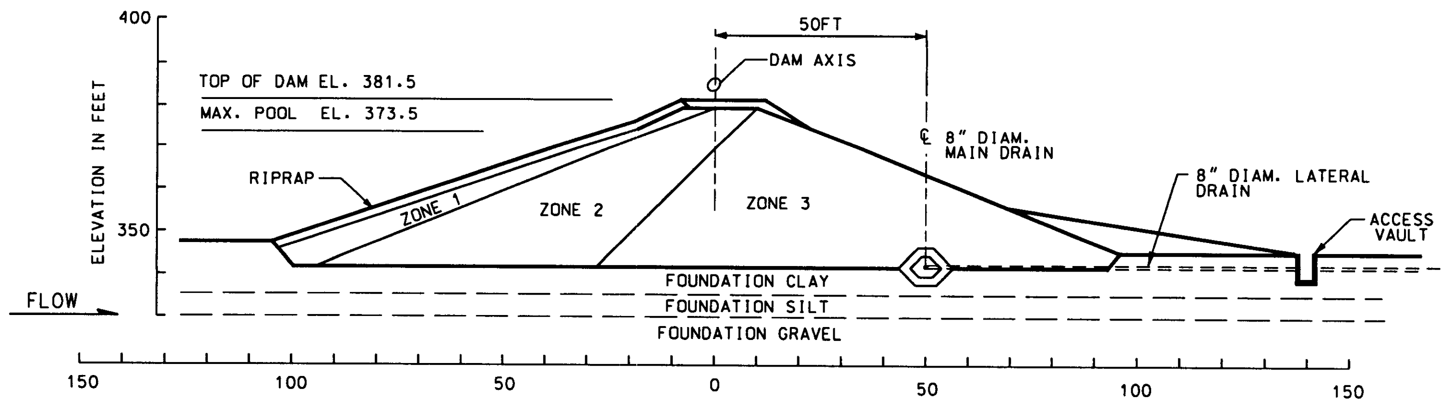
a. Location and Description of Project. Fern Ridge Dam is located on the Long Tom River in Lane County, Oregon, 12 miles northwest of the city of Eugene. Fern Ridge Dam is primarily a flood control dam. The project also provides significant recreation, irrigation, and environmental benefits. The main dam is a zoned, rolled earth embankment with a maximum height of 44 ft and a total length of 6,610 ft including the concrete spillway section.

b. General Description of Embankment, Foundation, and Drainage System. Figure 1.02-1 shows the embankment section at station 49 + 40. A layer of riprap two feet thick was placed on the upstream slope. Zone 1 is composed of pervious gravels. Zone 2 is the impervious zone. It is composed of alluvial silty clay with sand and gravel, with layers of predominantly granular soil. Zone 3 was to be constructed of random material. The specification stated, "In general, this material shall be placed so that the more impervious materials are placed next to Zone 2 and graded to pervious towards the downstream edge of the section." Laboratory tests performed on Zone 3 material in 1980 showed the material to have the same properties as those of Zone 2. A disposal zone sits at the downstream toe of the embankment and appears to be consistent with Zone 3 material properties.

The foundation beneath the embankment consists of a low permeability clay layer, silty sand layer, and gravel foundation. The 2 to 12 ft of foundation clay extends upstream of the dam into the reservoir. The foundation silty sand ranges in thickness from 0 to 9 ft and was deemed potentially liquefiable by the Phase I Seismic Stability Evaluation that was completed in September 2002. The foundation gravel, a clayey gravel, ranges in thickness from 80 to 100 ft.

Seepage through the earthen embankment is controlled and collected by the corrugated metal pipe drainage system. The drainage system has a longitudinal main drain running the length of the dam that is offset 50 feet downstream of the dam centerline. Lateral drains are located roughly every 500 feet along the length of the dam. Figure 1.02-2 shows the drainage system in plan view. Figure 1.02-1 shows a section of the main drain and a section of a lateral drain. The main drain is an 8 inch, perforated, corrugated metal pipe with bituminous coating. The filter around the pipe has two zones: $^{3}/_{8}$ inch to 2 inch washed gravel surrounding the pipe and selected borrow pit gravels surrounding the washed gravel. The lateral drains are similar to the main drain except the lateral drains are not perforated and the filter only surrounds the top half of the lateral pipes; the bottom portion of the lateral pipe sits directly on the foundation.

The drainage system is 60 years old. The service life of corrugated metal pipe is approximately 25 to 30 years; the service life of bituminous coatings is usually only 1 or 2 years (Ohio Department of Natural Resources, 2003).



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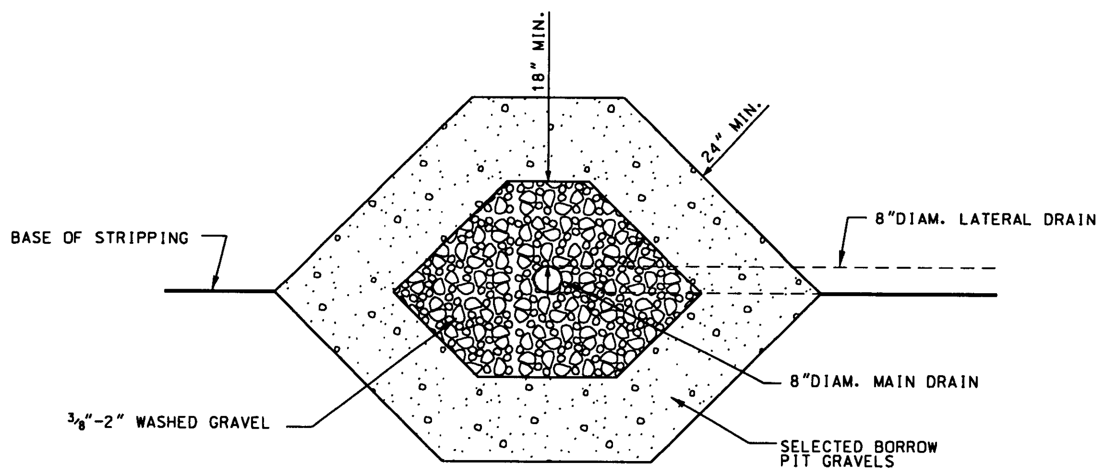
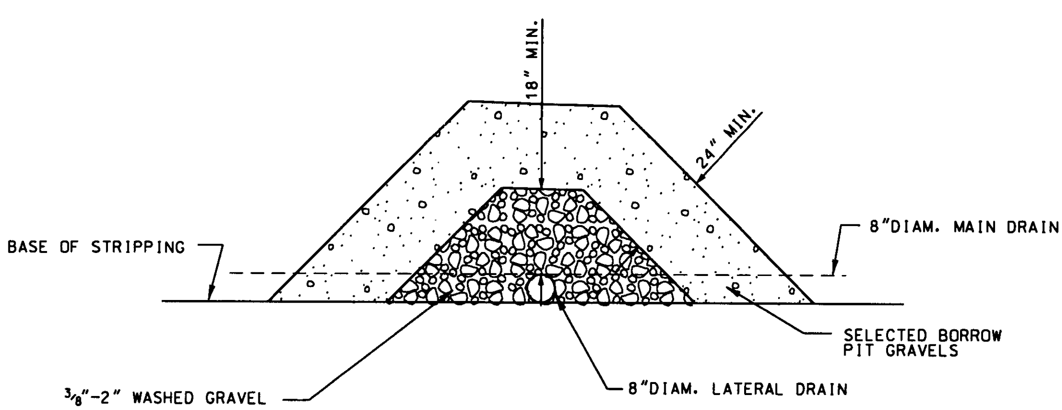


MAIN DRAIN DETAIL



LATERAL DRAIN DETAIL

Figure 1.02-1 Typical Dam Cross Section and Details of Main and Lateral Drains

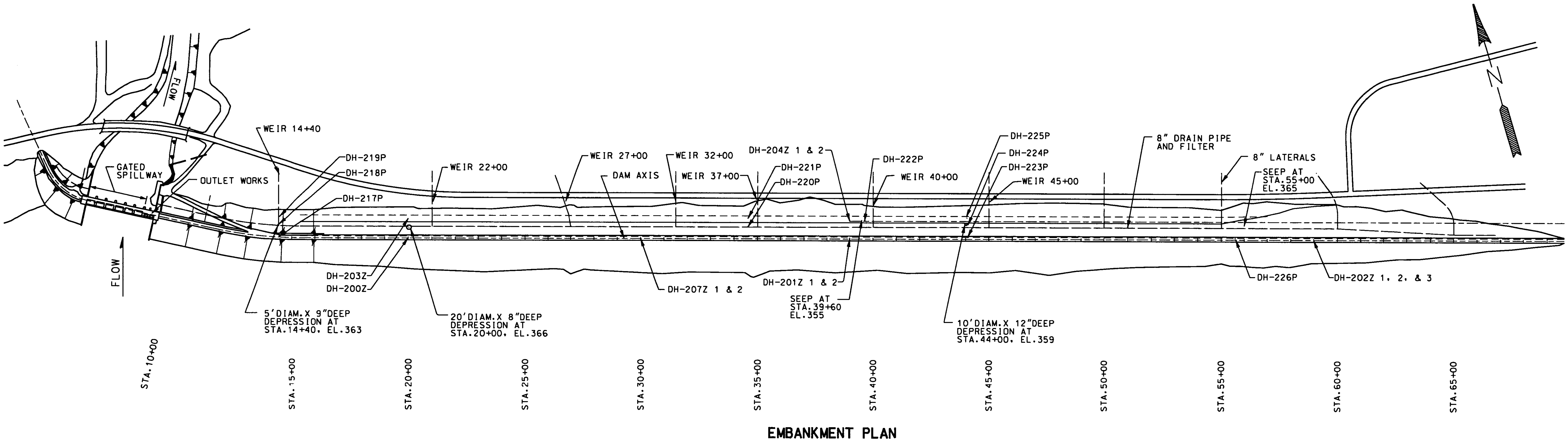


Figure 1.02-2 Plan View of Embankment

SECTION II – OBSERVATIONS OF DISTRESS

2.01. Depressions on Downstream Slope of Embankment. Three bowl-shaped depressions have been found on the downstream slope of the embankment. Table 2.01-1 shows the dates the depressions were discovered, the locations of the depressions, and the approximate size of the depressions. The first observed depression (Station 44) was discovered when the Project started mowing the embankment in August 2002. Mowing had previously been contracted out. The last two depressions were observed during visual inspections of the downstream embankment slope.

Table 2.01-1. Depressions on downstream slope of embankment.

Date of discovery	Station	Offset from dam centerline (feet)	Approximate size
August 2002	44	61	10 ft diameter, 12 in. deep
February 2003	20	43	20 ft diameter, 8 in. deep
February 2003	14+40	50	5 ft diameter, 8 in. deep

Survey markers were placed in the bottoms of the depressions at stations 44 and 20. As of May 2003, measured changes in survey marker elevations have been insignificant (less than 0.05 feet).

2.02. Inspection of Lateral Drains. The lateral drain pipes were inspected with a video rover on September 19, 2002. Prior to inspection with the rover, sediment was flushed out of the pipes using a hydrojet. The sediment flushed from the pipes included silt, sand, gravel, and roots. Some of the pipes needed to be flushed several times to allow the rover to advance in the pipes.

Only the lateral drain pipes were inspected with the video rover. The rover was not able to turn from the lateral drains into the main drain, so the main drain was not inspected.

The video showed that the lateral drain pipes are severely deteriorated in isolated places. Corrosion of the metal has led to scaling and the development of holes and open strips in the pipes. Even after flushing the lateral drains with the hydrojet, some sections had large amounts of silt/clay and gravel. In some places the thickness of silt in the bottom of the pipe was 4 inches. The gravel ranged in size up to 2 inches in diameter and is presumably from the filter material around the pipes.

The notes below from the geologist observing the inspection show the severity of the pipe deterioration. The notes are for the lateral drains at stations 22 and 14+40. Only the entrances of the pipes were videoed at these two stations.

Station 22: "Full of rocks, jagged edge visible, do not want to wash further for fear of mining embankment material. Sides of pipe rusted through at water line, rocks still on bottom and may indicate that the bottom of the drainpipe is holed or missing. ROV [rover] could not negotiate rocks, abandoned attempt."

Station 14+40: "Began with rewash based upon previous report." "Washed to Tee @ 90-95', flushed front to back 5-6 times and made some progress cleaning pipe but effort removing too much gray silt, therefore determined that continuation may be detrimental. Attempted ROV [rover] but getting stuck in rocks and silt."

2.03. Increases in Lateral Drain Flow Rates During Heavy Winter/Spring Rain Events. During heavy rain events, the flow rates from the lateral drains increase within one day by up to 10 gpm.

Different kinds of material have been observed in the discharge water from the lateral drains. Table 2.03-1 shows descriptions of materials collected from the lateral drains at five stations during heavy rain events.

Table 2.03-1. Materials observed in lateral drain discharge water during heavy rain events.

Station	Material description
27	silty sand
32	organics: brown fibers, orange flakes
37	organics: brown fibers, orange flakes; sand grains
40	organics: brown fibers, orange flakes
50	black particles of bituminous coating from the pipe

2.04. Seepage Emerging from the Downstream Slope. Seepage was observed emerging from the downstream slope of the embankment in two locations. Seepage was first observed in February 2003 at station 39+60 at approximately elevation 355 ft, where the downstream slope transitions from 2.5H:1V to 6.5H:1V. The highest elevation of observed seepage at station 39+60 was approximately 360 ft. In early May 2003, seepage was observed at station 56 at approximately elevation 365 ft. The cross-section at station 56 has different side slopes than the cross-section in Figure 1.02-1. In addition, the foundation elevation rises at the east end of the dam, resulting in a smaller, but wider, cross-section.

There is a lack of reliable embankment piezometric water level data corresponding to the conditions of seepage emerging from the embankment. In February 2003, a vibrating wire pressure sensor was installed in one of the five embankment piezometers installed in 1980. Data from the sensor showed the water level in the piezometer rose to the top of the standpipe during heavy rain events. It was discovered that rainwater was pooling around the piezometer monument casing and spilling into the piezometer standpipe. It takes several days for the water level in the piezometer to return to equilibrium due to the low permeability of the embankment soil. Rainwater might have been spilling into the other four embankment piezometers, too, resulting in unreliable piezometer readings. Tall monument casings are being installed to these piezometers to prevent rainwater from influencing the readings.

In late May 2003, after the reservoir was filled, seepage stopped emerging from these two locations and the saturated area around each location dried up. It was concluded that the seepage emerging from the slope was related to rainfall in the winter and spring, not to seepage through the embankment from the reservoir.

2.05. Concerns. The features of distress described above have led to two main concerns: a) internal erosion into the failing drainage system and b) downstream slope instability.

a. Internal Erosion into the Failing Drainage System. The drainage system was designed to collect seepage through the dam while restraining soil particles from eroding into the drain. There is concern that the failing drainage system is losing its ability to restrain soil particles. If internal erosion is allowed to progress, large voids may form, increasing the seepage through the embankment, and threatening the stability of the dam.

b. Downstream Slope Instability. Slope instability is a concern in winter/spring because of the seepage emerging from the downstream slope of the embankment. There is concern that the water level in the embankment in winter/spring is higher than the water level assumed in analyzing the long-term stability of the dam during design. A higher water level results in lower soil strength, decreasing embankment stability.

The investigations and instrumentation described in sections III and IV were designed to gather the information necessary to understand the causes of the observed distress, operate the dam safely, and design repair alternatives.

SECTION III – INVESTIGATIONS

3.01. Investigation Completed during Spring/Summer 2003. A geotechnical investigation was performed by Cornforth Consultants, Inc. to 1) look for voids in the embankment and foundation beneath the depressions, 2) determine soil properties needed for analysis of current conditions and design of repair alternatives, 3) determine piezometric water levels in the embankment, and 4) prepare a Geotechnical Data Report. The investigation included drilling and soil sampling, installation of piezometers in the embankment, in-situ permeability tests in the foundation, and laboratory tests for material properties. Below is a summary of key information from Cornforth's 2003 Geotechnical Data Report: Fern Ridge Dam Embankment Drain Investigation.

a. Results of Drilling and Soil Sampling. The ten drilling locations (DH-217 to DH-226) are shown on Figure 1.02-2. DH-218 and DH-224 were located in the depressions at stations 14+40 and 44+00. DH-222 was located at station 39+60 where seepage was observed in the winter and spring. Two holes were drilled at each location: 1) one using the mud-rotary technique with Standard Penetration Tests (SPTs) at 2.5 ft intervals and 2) one using a hollow-stem auger for collecting relatively undisturbed soil samples and installing a piezometer.

Soft soil with SPT blow counts from 0 to 1 was encountered in the top 7.5 ft of DH-218 in the depression at station 14+40. Wood was encountered from 4.5 to 5 ft depth in DH-224 in the depression at station 44+00. These subsurface conditions were only observed in these borings. No voids were encountered in the embankment and foundation.

Drain filter material was encountered in DH-218, DH-219, and DH-220. The material generally consisted of medium dense gravel with silty clay and sand.

Before installing piezometers in the hollow-stem auger borings, in-situ falling head permeability tests were performed in the foundation sands and gravels. The average permeability value from the tests was 7×10^{-4} cm/s, which is in the expected range of values.

<u>b. Piezometer Installation.</u> One open-standpipe piezometer was installed in each hollow-stem auger boring after the in-situ permeability test was performed. Each piezometer is a one-inch-diameter PVC tube with the slotted section and surrounding filter sand located near the bottom of the embankment. The piezometers are used to measure water levels in the embankment.

<u>c. Laboratory Testing.</u> Laboratory tests were performed to determine the following material properties: moisture content, Atterberg limits, grain-size distribution, undrained shear strength, and effective stress shear strength envelope for the embankment. Moisture contents were determined on all SPT samples. Atterberg limits and grain-size distributions were determined for 14 samples (embankment and foundation). This information was used for soil classification and will be used to evaluate filter criteria for current conditions and repair alternatives. The undrained shear strength of the embankment soil is needed to design repairs to the embankment. Undrained shear strength was measured in 9 unconfined compression tests. The average value was 1300 psf. The effective stress shear strength envelope for the embankment was determined and will be used to analyze the current stability of the downstream slope of the embankment.

<u>3.02. Investigations Planned for Fall/Winter 2003.</u> Four investigation tasks are planned for Fall/Winter 2003: 1) piezometer installations, 2) sprinkler tests, 3) test pits, and 4) video inspection of the main drain.

Eighteen open-standpipe piezometers will be installed by early October 2003, including 10 terminated in the embankment and 8 terminated in the foundation.

Three sprinkler tests will be performed in mid-October 2003 to simulate rainfall and see how rainfall infiltration affects the downstream slope of the embankment. The primary goals of the tests are to 1) measure the water levels in the embankment caused by rainfall infiltration, 2) measure the strength of the embankment soil after it has softened due to rainfall infiltration, and 3) measure how much rainfall infiltrates down to the drainage system, and how fast it infiltrates.

The sprinkler tests will be performed at stations 14+40, 34+00, and 40+00. A system of sprinklers will be set up at each station to apply water to the embankment for 4 weeks. Water levels in the embankment will be measured during the tests using deep open-standpipe piezometers and shallow drive-point piezometers. The amount of water infiltrating into the embankment and coming out the drainage system will be measured during the tests. After the tests, samples of the softened embankment soil will be taken and the strength of the samples will be measured.

Test pits will be dug at the three depressions described in section II, 2.01. The goals are to investigate the causes of the depressions and determine the extent of soft soil beneath the depressions. In addition, shallow pits will be dug into the drain filter material near the ends of the lateral drains at stations 14+40 and 45+00. The drain filter material will be evaluated for its suitability as filter material using gradation and Atterberg limits test results.

The main drain will be inspected with a 12-ft-long articulated video rover capable of carrying 5,000 ft of cable. The rover will enter the main drain at the east end of the embankment. If an existing pipe collapse prevents the rover from passing, a vertical shaft *may* be constructed to access the pipe on the other side of the collapse to continue the inspection. It is desirable to inspect the entire main drain to look for large voids adjacent to the main drain. Installation of a shaft may also be used to access the drain for repair purposes.

SECTION IV – INSTRUMENTATION

4.01. Weirs for Lateral Drains. Weir plates with 60 degree V-notches were installed downstream of the lateral drains at stations 14+40, 22, 27, 32, 37, 40, and 45. Both pressure transducers and staff gages were installed behind the weirs to measure the head over the weir crests. The pressure transducers are connected to the Supervisory Control and Data Acquisition (SCADA) system and the District's radio network. Both head and flow rate for each weir can be monitored every 30 minutes at Lookout Point and the Portland District office to watch for increases due to internal erosion.

Flow rates have been relatively steady during Summer 2003. Table 4.01-1 shows the range of flow rates for all lateral drains, including the east drains 50, 55, 60, and 65 that do not have weirs.

Table 4.01-1. Summer 2003 lateral drain flow rates and stages.

Lateral drain station	Flow rate (gpm)	Stage (ft)
14+40	0.8 – 1.2	0.07 – 0.08
22	1.6 – 3.9	0.09 – 0.13
27	0.1 – 0.8	0.03 – 0.07
32	0.6 – 1.2	0.06 – 0.08
37	0	0
40	0.1 – 1.6	0.03 – 0.09
45	7 – 10	0.16 – 0.19
50	0.25 – 0.75	
55	0 – 0.25	
60	0.25 – 2.0	
65	0	

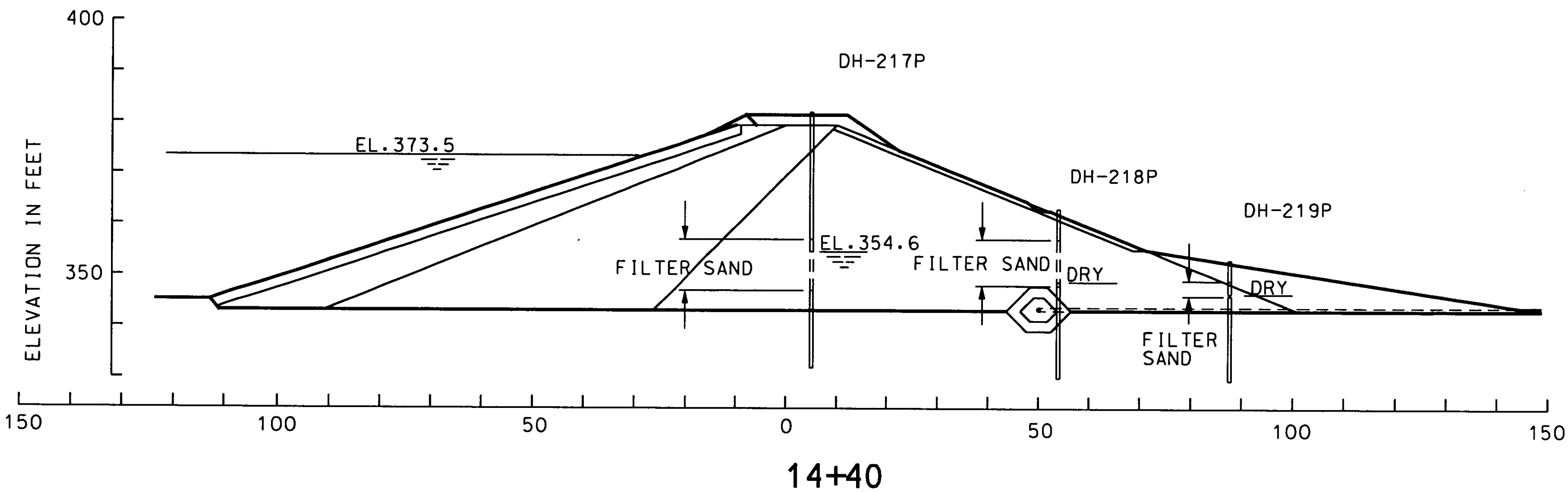
Material accumulating in the boxes behind the weirs is collected periodically. During Summer 2003 a significant amount of organic material has come out some of the lateral drains.

4.02. Pressure Transducers for Embankment Piezometers. Pressure transducers were installed in the 10 new embankment piezometers described in section III, 3.01. The transducers are not connected to SCADA or the radio network; Portland District staff travel to the project weekly to download the data. The data is reviewed weekly by Portland District staff.

4.03. Summer 2003 Piezometric Water Levels. Figures 4.03-1 through 4.03-5 show Summer 2003 water levels measured in the 11 piezometers installed in 1980 and the 10 piezometers installed in June 2003.

The summer piezometric water levels are generally reasonable for seepage through the dam from the reservoir. No seepage is emerging from the downstream slope.

4.04. Planned Instrumentation. After the 18 open-standpipe piezometers are installed in Fall 2003, there will be 38 open-standpipe piezometers total in the embankment and foundation. These include piezometers installed in 1980, June 2003, and Fall 2003. All 38 piezometers will have pressure transducers connected to the SCADA system and the District's radio network. Piezometric water level data will be monitored at Lookout Point and the Portland District office. **This automated instrumentation data is a critical component of the interim operating plan.**



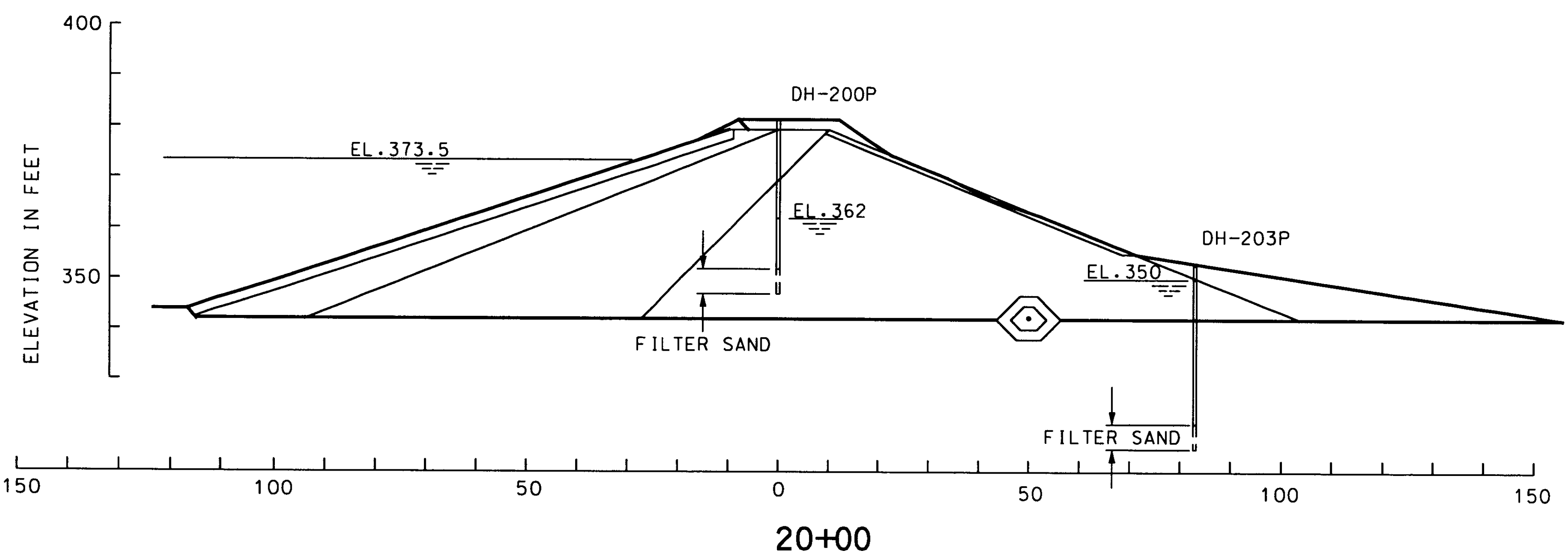
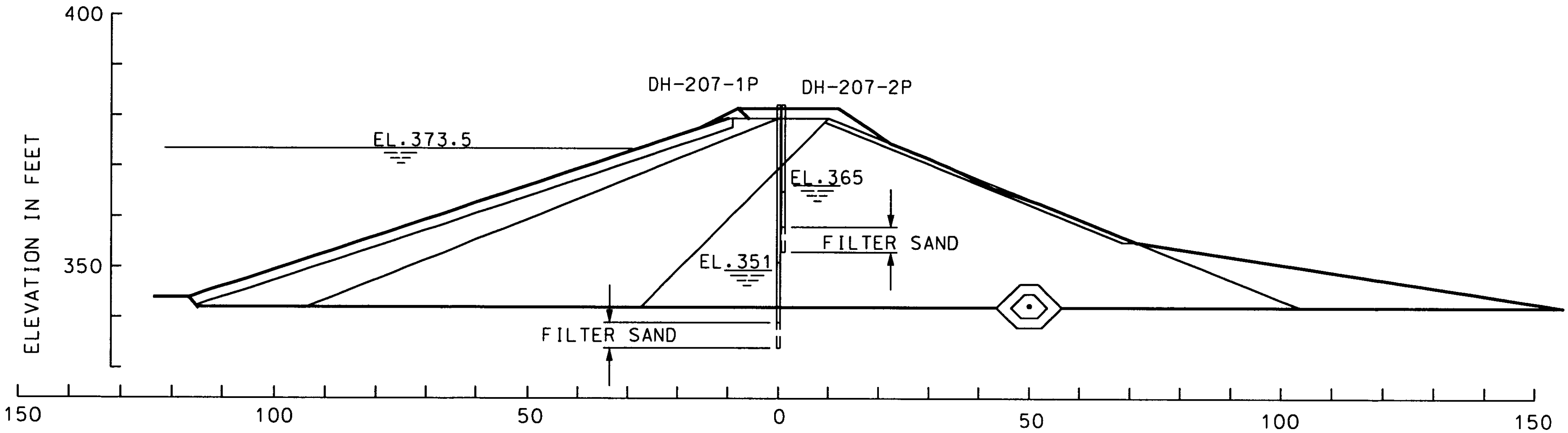
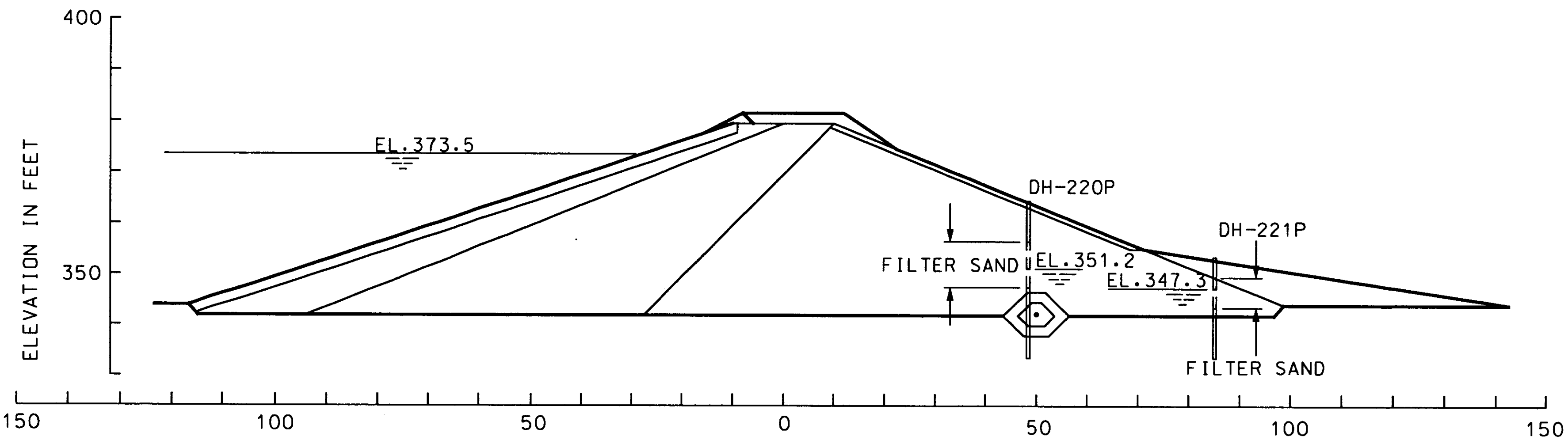


Figure 4.03-1. Summer 2003 Piezometric Levels at Stations 14+40 and 20+00

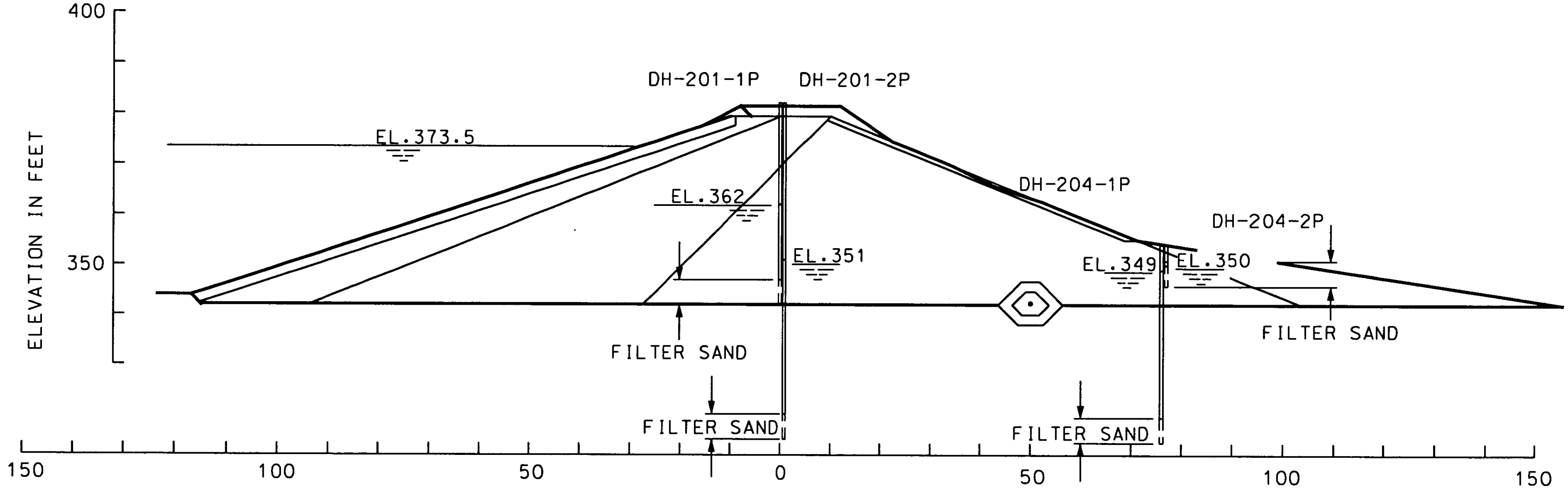


30+00

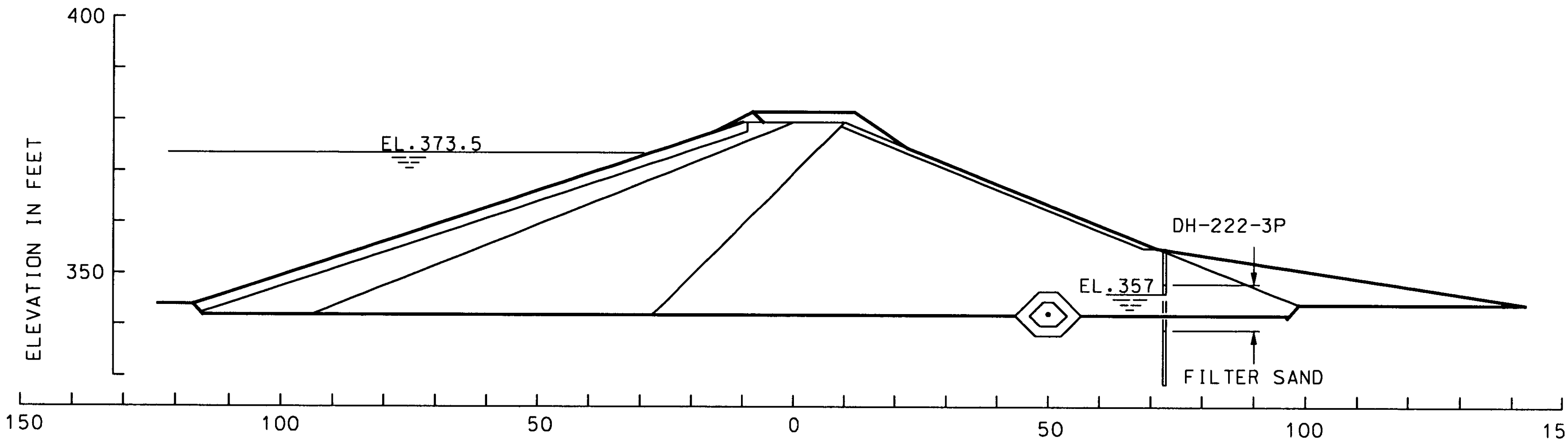


34+60

Figure 4.03-2. Summer 2003 Piezometric Levels at Stations 30+00 and 34+60



39+00



39+59

Figure 403-3. Summer 2003 Piesometric Levels at Stations 39+00 and 39+50

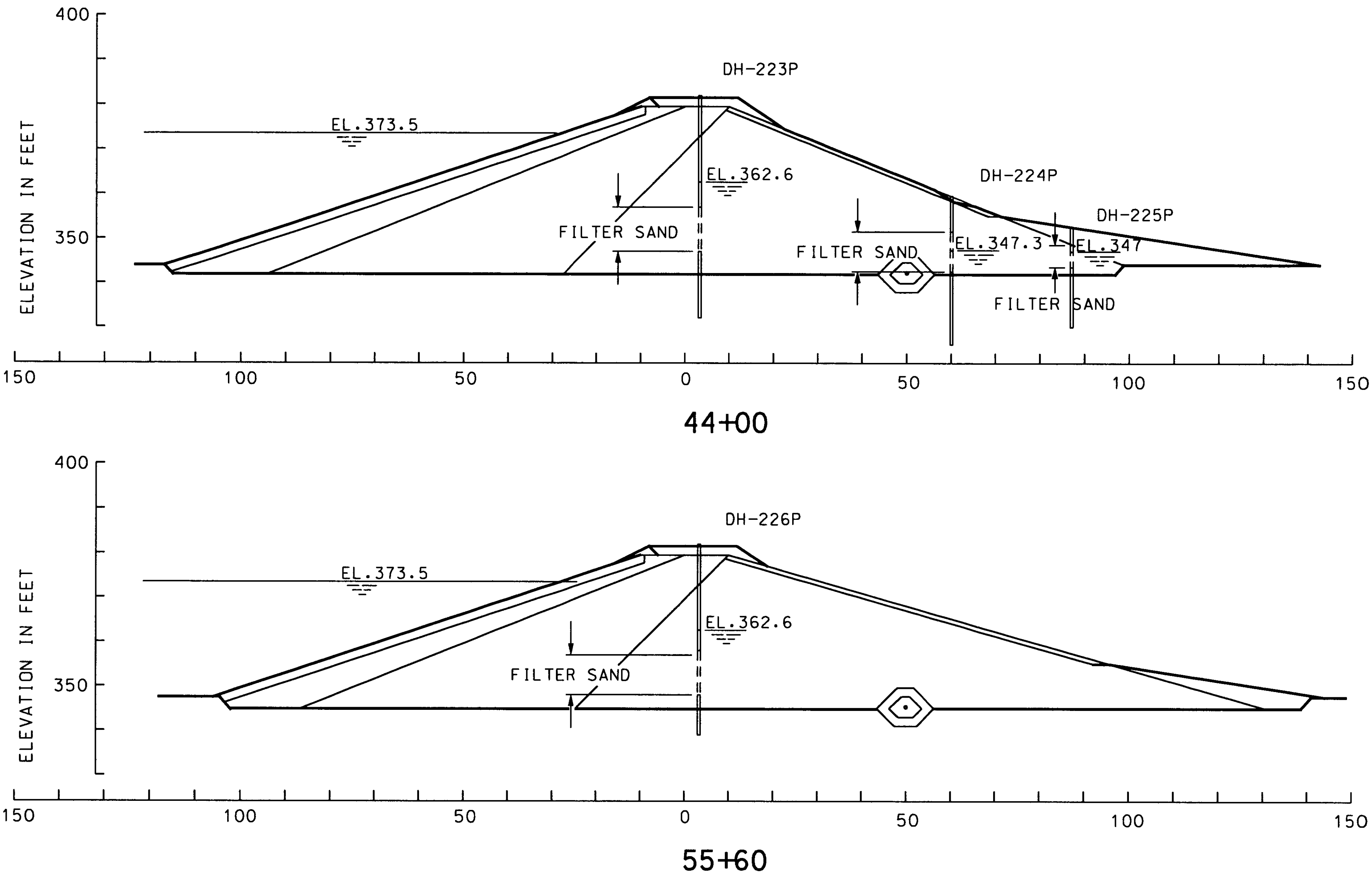


Figure 4.03-4. Summer 2003 Piezometer Levels at Stations 44+00 and 55+60

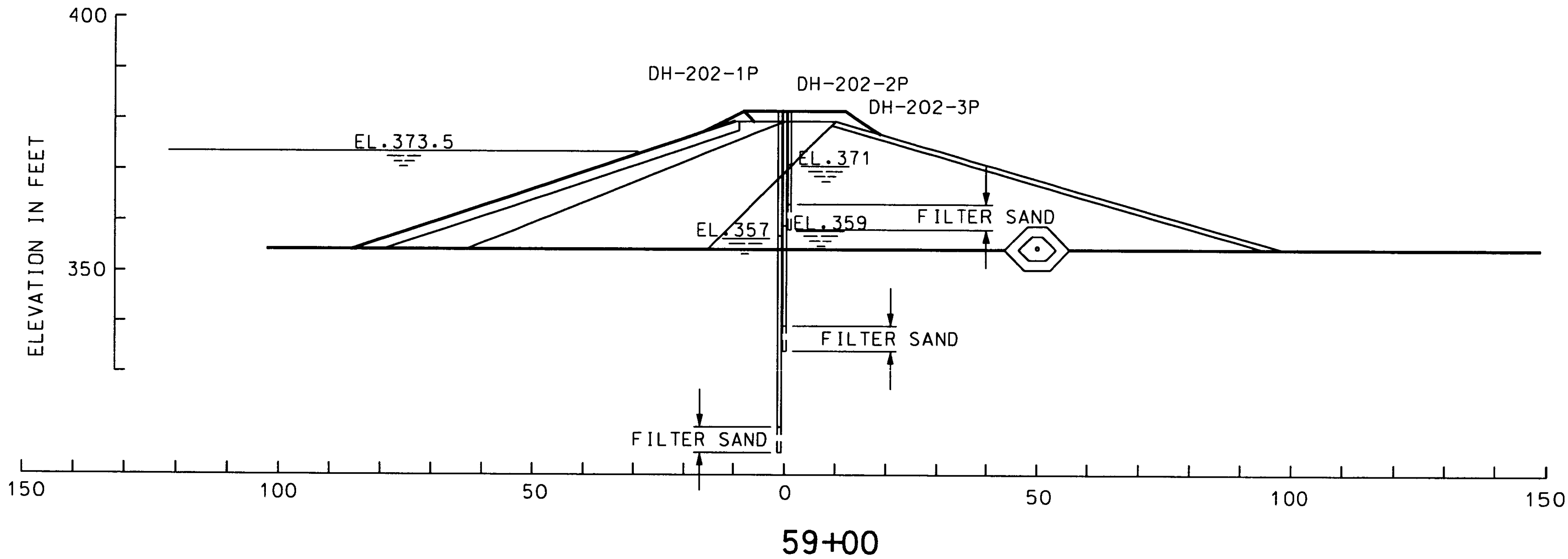


Figure 4.03-5. Summer 2003 Piesometric Levels at Station 59+00

SECTION V – INTERIM PLAN FOR OPERATING THE PROJECT

Until funding is available to fully repair the embankment, the project will be operated using a conditional operating plan. The conditional operating plan is based on close monitoring of piezometric water levels in the embankment to monitor downstream slope instability. In addition, an event alert system will be in place to prevent the initiation/progression of internal erosion into the failing drainage system. Data flow communication for the conditional operating plan and the event alert system will be from the Portland District Geotechnical Design Section to the Portland District Reservoir Regulation and Water Quality Section to the Northwestern Division Reservoir Control Center.

5.01. Conditional Operating Plan. The maximum reservoir elevation for flood control will depend on the piezometric water levels in the embankment. For example, consider a heavy rain event occurring in mid-December. If piezometric water levels in the embankment are low, the reservoir will be allowed to fill to the established maximum flood control pool elevation of 375.1 ft. However, if piezometric water levels in the embankment are high, the reservoir will only be allowed to fill to elevation 371 ft for flood control. The establishing of elevation 371 ft as the critical reservoir elevation for downstream slope stability with high piezometric water levels is discussed below.

Reservoir filling will start February 1st, as normal. The normal reservoir filling schedule will be followed until the reservoir reaches elevation 371 ft, typically near the end of March. If piezometric water levels in the embankment are low, the reservoir will continue following the normal filling schedule to the maximum conservation pool elevation of 373.5 ft. If piezometric water levels are too high, the reservoir will be held at elevation 371 ft.

The conditional operating plan may be altered in the future if additional information indicates conditions safer than originally thought, or if future measures are implemented to improve drainage and stability.

The critical reservoir elevation of 371 ft is based on a preliminary stability analysis with an assumed phreatic surface in the embankment during the rainy season. The phreatic surface in the embankment was assumed to be a straight line between the reservoir elevation and the highest observed elevation of seepage emerging from the downstream slope, 360 ft. This approximation assumes the phreatic surface due to seepage from the reservoir is added to by rainwater infiltration as shown in Figure 5-1. Figure 5-2 shows safe and unsafe phreatic surfaces with seepage emerging from the downstream slope at elevation 360 ft. The sprinkler tests described in section III, 3.02 will provide measurements of the phreatic surface caused by simulated rainfall infiltration. This measured phreatic surface will be used in the stability analysis included in the future letter report. Another measured output of the sprinkler tests will be the strength of the embankment softened by the simulated rainfall infiltration, which will also be included in the future stability analysis. This measured softened strength is expected to be higher than the current assumed embankment strength used in the preliminary stability analysis. The results of the future stability analysis may change the plan for operating the project.

If the future stability analysis predicts unsafe conditions due to high piezometric levels during the rainy season, an interim measure may be designed and constructed to improve the stability of the downstream slope. Examples of possible interim measures are 1) installing a shallow drainage system on the downstream slope, 2) placing a granular berm on the downstream slope, and 3) covering the downstream slope with a membrane to prevent rainwater infiltration.

5.02. Event Alert System. Measurement of drain flow rates and visual observations will be used to watch for the initiation/progression of internal erosion into the failing drainage system. Drain flow rates from the pressure transducers behind the drain weirs will be monitored at Lookout Point and the Portland District office. Drain flow rates will also be periodically measured by hand by Fern Ridge project staff to verify the data from the pressure transducers. The event alert system will notify the appropriate personnel if there is a significant or unusual increase in flow rate indicating initiation/progression of internal erosion.

Figure 5-1. Influence of rainwater infiltration assumed in preliminary stability analysis.

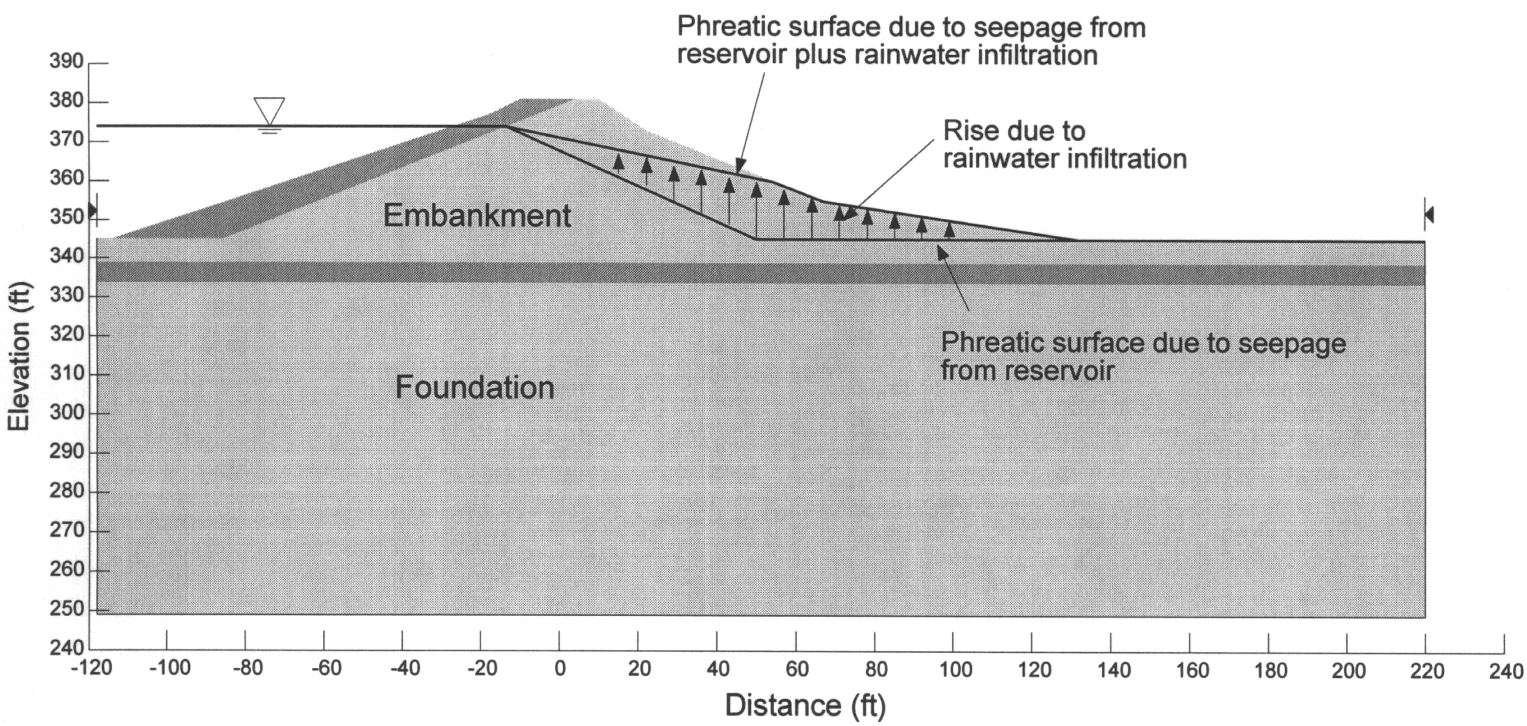
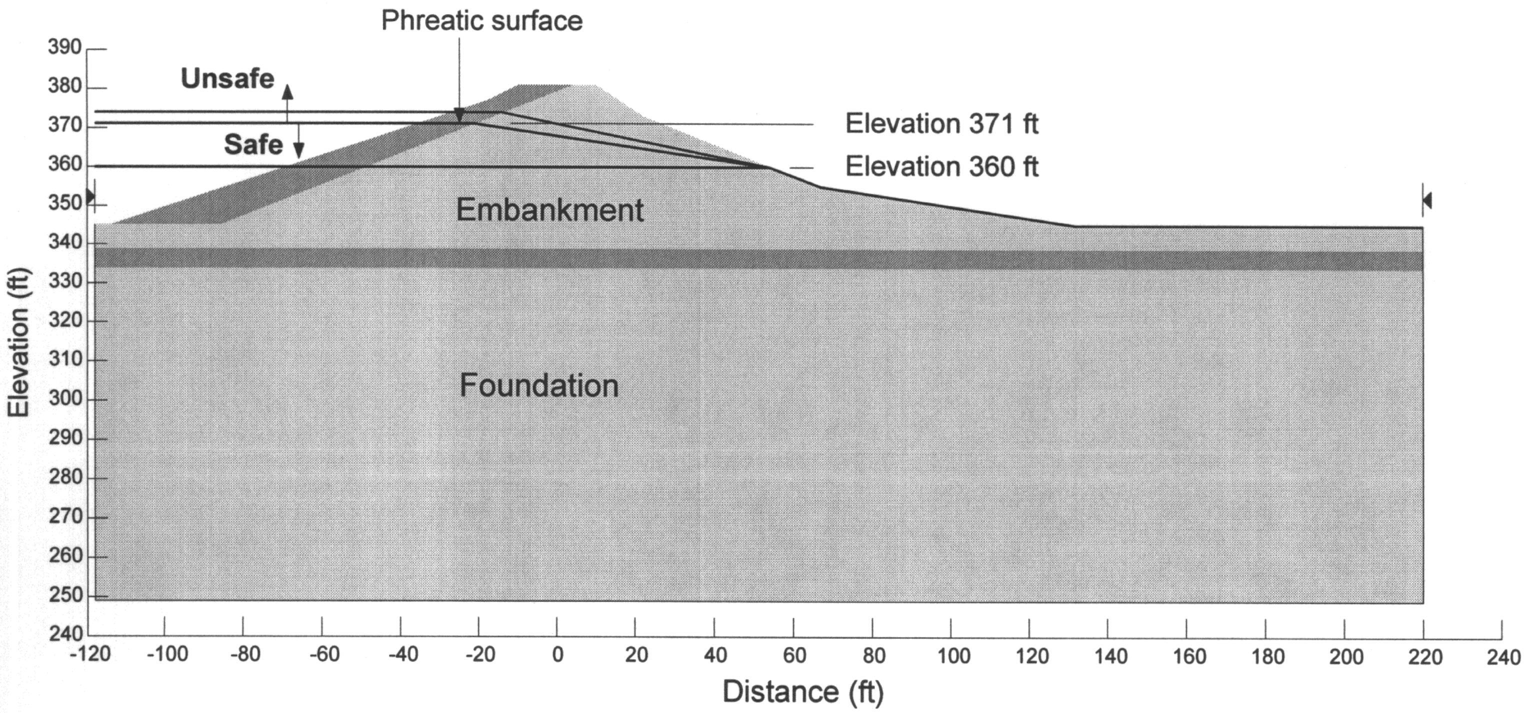


Figure 5-2. Safe and unsafe phreatic surfaces in embankment for downstream slope stability (preliminary analysis) with seepage emerging at elevation 360 ft.



SECTION VI – SUMMARY

Several features of distress have recently been observed at Fern Ridge Dam. Three depressions exist on the downstream slope. The 60-year-old drainage system is failing, initiated by corrosion of the corrugated metal pipe. Flow rates from the drainage system increase during heavy rain events. During the rainy season, seepage was observed emerging from the downstream slope of the embankment. These features of distress have led to two primary concerns: 1) internal erosion into the failing drainage system and 2) downstream slope instability.

Borings were made into two of the depressions in May/June 2003. Soft soil was found beneath the depressions. Test pits will be dug in Fall/Winter 2003 to investigate the extent of the soft soil and to look for voids.

Video inspection of the lateral drains in September 2002 showed severe deterioration in some places. The main drain will be inspected in Fall/Winter 2003.

Weirs and pressure transducers were installed at the outlets of 7 of the lateral drains. Measured flow rates can be monitored continuously at Lookout Point and the Portland District office to watch for increases due to internal erosion into the drainage system.

By October, 2003, thirty-eight piezometers with pressure transducers will be installed to measure water levels in the dam. This information is important for evaluating the stability of the downstream slope of the embankment. Sprinkler tests will be performed in Fall 2003 to simulate rainfall and the condition of seepage emerging from the embankment slope. Measurements of piezometric water levels and the strength of the softened embankment soil during the sprinkler tests will improve the evaluation of the downstream slope stability during the rainy seasons.

The interim plan for operating the project involves a conditional operating plan for the reservoir elevation and an event alert system to prevent the initiation/progression of internal erosion into the failing drainage system. The conditional operating plan may be altered in the future if further investigations indicate more favorable conditions, or if interim measures are constructed to improve downstream slope stability.

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